Appendix E
Design Examples

E-1. Package Plant Extended Aeration

Design an extended aeration package plant (prefabricated or pre-engineered) to treat a municipal wastewater flow of 125 000 L/d (33 000 gal/d). Solids retention typically ranges from 20 to 30 days; MLSS varies between 3 000 and 6 000 mg/L. The Food to Microorganism ratio (F/M) typically varies between 0.05 to 0.30. Influent BOD_5 and TSS will generally be about 250 mg/L. The dissolved oxygen (DO) concentration is in the 1.5 to 2.5 mg/L range and preferably will never be below 2.0 mg/L. Coarse bubble aerators will be used. Detention time in the aeration tank will be one day.

Table E-1
Design Assumptions

Influent/Effluent Composition - Given

Parameter	Influent	Effluent
BOD_5	250 mg/L	20 mg/L
TSS	250 mg/L	20 mg/L
TKN	40 mg/L	5 mg/L
Assumptions		
Minimum operating temperature:		17°C (62°F)
Site elevation above sea level:		137 m (450 ft)
Net sludge yield (kg MLSS/kg BOD_5):		0.76
DO mixed liquor concentration (C_O)		2 mg/L
Oxygen coefficients:		
kg O_2/kg BOD_5		1.28
kg O_2/kg NH_3-N		4.60
Transfer factors:		
α (typical for coarse bubble diffuser)		0.85
β (typical for domestic wastewater)		0.95
Sludge settling zone overflow rate:		< 10 $m^3/m^2/d$
Aeration tank detention time:		1 day
Typical O_2 transfer rate for coarse bubble diffusers:		30 kgO_2/kW-d (48 lbO_2/hp-d)
Solids retention:		25 days

a. Sludge production. Calculate the sludge production rate based on the desired BOD_5 removal. Calculate the system total solids mass based on the sludge production rate and the assumed solids retention time, as shown in Table E-2.

b. Aeration power. Calculate the blower capacity based on the sludge production rate, desired TKN removal/synthesis, and the site specific conditions, as shown in Table E-2.

c. Unit dimensions. Estimate the required unit process dimemsions including the chlorine contact tank based on one-day hydraulic detention time and using two sludge settling hoppers, as shown in Table E-2.

Table E-2
Package Plant Extended Aeration Design Calculations

a. Aerobic Volume

$$BOD_5\ Removed\ (kg/d) = \frac{Flow\ (L/d)}{10^6\ (mg/kg)} \times (BOD_{influent} - BOD_{effluent})\ (mg/L)$$

$$BOD_5\ Removed = \frac{125 \times 10^3}{10^6} \times (250 - 20) \cong 29\ kg/d\ (64\ lb/d)$$

$$Sludge\ Production\ (kg/d) = Net\ Sludge\ Yield\ (kg\ MLSS/kg\ BOD_5) \times BOD_5\ Removed\ (kg/d)$$

$$Sludge\ Production = 0.76 \times 29 = 22\ kg/d\ (48.5\ lb/d)$$

$$System\ Mass\ (kg) = Sludge\ Production\ (kg/d) \times Solids\ Retention\ (d)$$

$$System\ Mass = 22\ (kg/d) \times 25\ (d) = 550\ kg\ (1\ 213\ lb)$$

b. Aeration Power

$$NH_3 - N_{oxidized} = TKN_{influent} - Synthesis\ N - TKN_{effluent}$$

$$Synthesis\ N = 5\%\ waste\ activated\ sludge\ of\ total\ daily\ sludge\ production$$

$$Synthesis\ N = 0.05 \times 22\ (kg/d) = 1.1\ kg/d\ (2.4\ lb/d)$$

$$Synthesis\ N\ (mg/L) = \frac{1.1\ (kg/d) \times 10^6\ (mg/kg)}{125 \times 10^3\ (L/d)} = 8.8\ mg/L$$

$$NH_3 - N_{oxidized} = 40\ (mg/L) - 8.8\ (mg/L) - 5\ (mg/L) = 26.2\ mg/L$$

$$NH_3{-}N\ (kg/d) = 26.2\ (mg/L) \times 10^{-6}\ (kg/mg) \times 125 \times 10^3\ (L/d) = 3.28\ kg/d\ (7.2\ lb/d)$$

$$\begin{aligned} AOR = {} & 1.28\ (kgO_2/kgBOD_5) \times Synthesis\ N\ (kgBOD_5/d) + \\ & 4.6\ (kgO_2/kg\ NH_3{-}N) \times NH_3{-}N_{oxidized}\ (kg/d) \end{aligned}$$

$$AOR = 1.28\ (kgO_2/kgBOD_5) \times 1.1\ (kgBOD_5/d) + 4.6\ (kgO_2/kg\ NH_3{-}N) \times 3.28\ (kgNH_3{-}N/d)$$

$$AOR = 16.5\ kgO_2/d\ (36.4\ lbO_2/d)$$

where:

AOR = Actual Oxygen Requirements (kg O_2/d)

$$SAOR = AOR \times \frac{C_S\ (mg/L) \times \Theta^{(20-T)}}{\alpha \times (\beta \times C_{SW} - C_O)}$$

where:

SAOR = Standard Actual Oxygen Requirements (kg O_2/d)
Θ (temperature correction factor) = 1.024

(Sheet 1 of 3)

Table E-2 (Continued)

C_S (O_2 saturation concentration at standard temperature and pressure) = 9.02 mg/L
C_{SW} = correction factor for elevation (i.e., 450 ft) = 9.02 - 0.0003 × elevation
C_{SW} = 9.02 - 0.0003 × 450 = 8.88 mg/L (<u>NOTE:</u> 0.0003 may be used as rule-of-thumb describing a 0.0003 mg/L rise/drop in DO saturation concentration per every foot of elevation increase/decrease.)
C_O = 2 mg/L
α = 0.85; β = 0.95; T = 17°C (62°F)

$$SAOR = 16.5\,(kgO_2/d) \times \frac{9.02\ (mg/L) \times 1.024^{(20-17)}}{0.85 \times [0.95 \times 8.88\ (mg/L) - 2.0\ (mg/L)]} = 29.2\ kgO_2/d\ (64.4\ lbO_2/d)$$

$$Motor\ Requirements\ (kW) = \frac{SAOR\ (kgO_2/d)}{O_2\ Transfer\ Rate\ (kgO_2/kW{-}d)}$$

$$Motor\ Requirements\ (kW) = \frac{29.2\ (kgO_2/d)}{30\ (kgO_2/kW{-}d)} = 1.0\ kW\ (1.3\ hp)$$

Since blowers typically have an efficiency of 50% or less, select 2 aerators with 3.73 kW (5 hp) motors.

c. Unit Dimensions

1. Aeration Tank Volume = 125 m^3 at one day hydraulic detention

   Assume tank dimensions based on values typical of extended aeration systems:
   Operating depth = 3.048 m (10 ft)
   Width = 2.895 m (9.5 ft)

$$Tank\ Length = \frac{Volume}{Width \times Depth} = \frac{125}{2.895 \times 3.048} = 14.2\ m\ (47\ ft)$$

2. Sludge settling zone overflow rate.To calculate the sludge settling zone overflow, assume two sludge settling hoppers each with a top dimension of 2.895 m (9.5 ft) and a bottom dimension of 0.304 m (1 ft).

   Surface area of sludge settling zone = 2 × 2.895 × 2.895 = 16.76 m^2 (180 ft^2)

$$Overflow\ Rate = \frac{125\ (m^3/d)}{16.76\ (m^2)} = 7.46\ m^3/m^2{-}d\ (24.5\ ft^3/ft^2{-}d)$$

   Assume settling height above hoppers = 0.3 m (1 ft)

   Depth of hopper = 3.048 m - 0.3 m = 2.75 m (9 ft)

$$Hopper\ Volume = \frac{1}{3}\left(A_1 + A_2 + \sqrt{A_1 \times A_2}\right) \times depth$$

$$Hopper\ Volume = \frac{1}{3}\left(2.895^2 + 0.304^2 + \sqrt{2.895^2 \times 0.304^2}\right) \times 2.75 = 8.56\ m^3\ (302\ ft^3)$$

   Total Hopper Volume = 8.56 m^3 x 2 = 17.1 m^3 (604 ft^3)

   Sludge holding tank shall be located at head of tank and shall equal volume of sludge hoppers.

   Width = 2.895 m (9.5 ft)
   Depth = 3.048 m (10 ft)

$$Holding\ Tank\ Length\ (m) = \frac{Total\ Hopper\ Volume\ (m^3)}{Width\ (m) \times Depth\ (m)}$$

$$Holding\ Tank\ Length = \frac{17.1\ m^3}{2.895\ m \times 3.048\ m} = 1.94\ m\ (6.4\ ft)$$

(Sheet 2 of 3)

Table E-2 (Concluded)

3. If chlorination is used as a disinfectant, the chlorine contact tank shall have a detention time of 75 min; therefore the tank shall have a capacity of 6.5 m^3 and the tank dimensions will be:

Width = 2.895 m (9.5 ft)
Length = 3.048 m (10 ft)

$$Chlorine\ Contact\ Tank\ Depth\ (m) = \frac{Volume\ (m^3)}{Width\ (m) \times Depth\ (m)} = \frac{6.5\ m^3}{2.895\ m \times 3.048\ m}$$

Chlorine Contact Tank Depth = 0.74 m (2.4 ft)

(Sheet 3 of 3)

d. Equipment specifications. Figure E-1 presents a plan view and a side view of the pre-engineered package plant extended aeration with the following specifications:

(1) The unit package plant will require no pre-treatment other than wastewater pumping from an influent manhole lift station.

(2) The influent pipe shall have a minimum of a 150 mm (6 in.) diameter from the influent manhole and will discharge directly to a combination comminutor/bar screen located ahead of (and on top of) the aeration tank.

(3) Two 3.73 kW (5 hp) blower assemblies shall provide air at 31 kPa (4.5 psi) to ensure a 2.0 mg/L DO level in the aeration tank at all times.

(4) A minimum of 44 diffusers will be required to distribute aeration at the aeration tank floor level. At least six (6) diffusers will be provided in the sludge holding tank and one (1) in the chlorine contact tank.

(5) A totalizing flow meter will be provided to record the daily flow patterns and total.

(6) A minimum of eight spray nozzles will be required on the top of the aeration tank on the side opposite to the aeration diffuser drops.

(7) Each sludge hopper will be equipped with an air lift pump with openings 150 mm (6 in.) above the hopper bottoms.

(8) The air lift pumps will discharge to a combination 75 mm (3 in.) sludge return and sludge waste line to the head of the tank.

(9) Blower units shall be controlled by a blower panel located above the aeration tank.

(10) Scum skimmers will be provided at a scum baffle ahead of the tank discharge (by V-notch weir) to the chlorine contact tank.

(11) Should ultraviolet disinfection be chosen in lieu of chlorination of tank effluent, an in-pipe rather than open channel effluent flow may be specified.

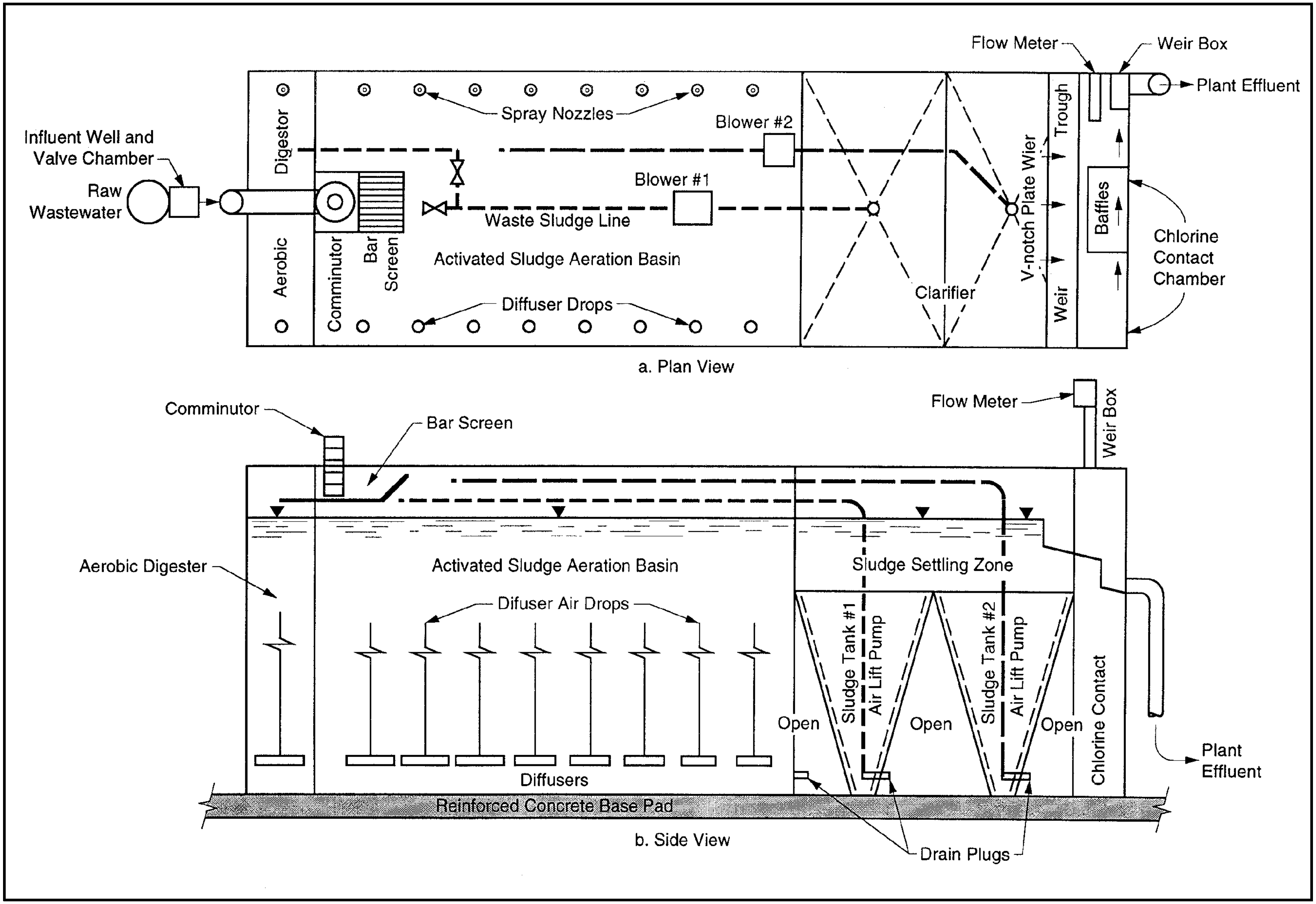

Figure E-1. Pre-engineered package plant extended aeration

E-2. Oxidation Ditch (Continuous-Loop Reactor) Carrousel—Wraparound

Design a carrousel (circular or wraparound) oxidation ditch to treat municipal wastewater at an average influent flow rate of 378 500 L/d (100,000 gal/d). The new system will use mechanical aerators and have the design parameters shown in Table E-3.

Table E-3
Design Parameters and Assumptions

Influent/Effluent Composition		
Parameter	Influent	Effluent
BOD_5	250 mg/L	5 mg/L
TSS	300 mg/L	10 mg/L
TKN	30 mg/L	5 mg/L
NH_3-N	--	0.5 mg/L
Assumptions		
Minimum wastewater temperature:		16°C (61°F)
Process solids retention time:		20 days
MLSS concentration:		4000 mg/L
Net yield (kg MLSS/ kg BOD_5):		0.76
Oxygen coefficients:		
kg O_2/kg BOD_5		1.28
kg O_2/kg NH_3-N		4.60
Transfer factors:		
α (typical for mechanical aerator)		0.90
β (typical for domestic wastewater)		0.95
Typical O_2 transfer rate for mechanical aerator:		
		37 kgO_2/kW-d
Site elevation (sea level + tank height):		(60 lbO_2/hp-d)
Clarifier overflow rate:		9.1 m (30 ft)
Side water depths,		16.3 $m^3/m^2/d$
clarifier and reactor channels:		3.04 m (10 ft)

a. Carrousel volume. Calculate the sludge production rate based on the desired BOD_5 removal. Calculate the system total solids mass based on the sludge production rate and the assumed solids retention time. Calculate the carrousel volume from the calculated system total solids mass and the assumed MLSS concentration, as shown in Table E-4.

b. Aeration power. Calculate the blower capacity based on the desired TKN removal/synthesis and the site specific conditions, as shown in Table E-4.

c. Clarifier diameter. Estimate the required wraparound clarifier diameter based on the assumed clarfier overflow rate and the side water depths, as shown in Table E-4.

d. Carrousel specifications. The carrousel shown in Figure E-2 has the following specifications:

Clarifier diameter:	6.4 m (21 ft)
Inner channel:	4 m (13 ft)
Outer channel:	4 m (13 ft)
Entire tank diameter:	14 m (46 ft)
Walls and miscellaneous equipment thickness:	2 m (6.5 ft)
Constructed carousel diameter:	14 m + 2 m = 16 m (52.5 ft)

Table E-4
Oxidation Ditch Design Calculations

a. Carrousel Volume

$$BOD_5\ Removed\ (kg/d) = \frac{Flow\ (L/d)}{10^6\ (mg/kg)} \times (BOD_{influent} - BOD_{effluent})\ (mg/L)$$

$$BOD_5\ Removed = \frac{378.5 \times 10^3}{10^6} \times (250 - 5) = 92.7\ kg/d\ (204\ lb/d)$$

$$Sludge\ Production\ (kg/d) = Net\ Yield\ (kg\ MLSS/kg\ BOD_5) \times BOD_5\ Removed\ (kg/d)$$

$$Sludge\ Production = 0.76 \times 92.7 = 70.5\ kg/d\ (156\ lb/d)$$

$$System\ Mass = 70.5\ (kg/d) \times 20\ (d) = 1\ 410\ kg\ (3\ 107\ lb)$$

$$Carrousel\ Volume\ (m^3) = \frac{System\ Mass\ (kg) \times 10^3}{MLSS\ Concentration\ (mg/L)}$$

$$Carrousel\ Volume = \frac{1.41 \times 10^3 \times 10^3}{4 \times 10^3} = 353\ m^3\ (12{,}466\ ft^3)$$

b. Aeration Power

Synthesis N =5% wasted activated sludge of total daily sludge production

Synthesis N = 0.05 × 70.5 (kg/d) = 3.52 kg/d (7.75 lb/d)

Synthethis N = 9.3 mg/L (for a daily flow of 378 500 L/d)

$NH_3 - N_{oxidized}$ *= TKN (mg/L) - Synthesis N (mg/L) -* $N\text{-}NH_{3(effluent)}$ *(mg/L)*

$NH_3 - N_{oxidized} = 30 - 9.3 - 0.5 = 20.2$ *mg/L*

$NH_3 - N_{oxidized} = 7.6$ *kg/d (16.9 lb/d) - based on a daily flow of 378 500 L/d*

$AOR = kg\ O_2/kg\ BOD_5 \times Synthesis\ N\ (kg\ BOD_5/d) + kg\ O_2/kg\ NH_3\text{-}N \times NH_3\text{-}N_{oxidized}\ (kg/d)$

$AOR = 1.28 \times 3.52 + 4.6 \times 7.6 = 4.5 + 35.0 = 39.5\ kg\ (87\ lb)$

where:
AOR = Actual Oxygen Requirements (kg O_2/d)

$$SAOR = AOR \times \frac{C_S\ (mg/L) \times \Theta^{(20-T)}}{\alpha \times (\beta \times C_{SW} - C_O)}$$

$$Clarifier\ Area\ (m^2) = \frac{Design\ Flow\ (m^3/d)}{Overflow\ Rate\ (m^3/m^2/d)}$$

where:

SAOR = Standard Actual Oxygen Requirements (kgO_2/d)
Θ (temperature correction factor) = 1.024
C_S (DO saturation concentration at standard temperature and pressure conditions) = 9.02 mg/L
C_{SW} = Correction factor for elevation (i.e., 30 ft) = 9.02 - 0.0003 × elevation
C_{SW} = 9.02 - 0.0003 × 30 = 9.011 mg/L (NOTE: 0.0003 may be used as rule-of-thumb describing a 0.0003 mg/L rise/drop in DO saturation concentration per every foot of elevation increase/decrease.)

(Sheet 1 of 3)

Table E-4 (Continued)

C_O = 2.0 mg/L
α = 0.90; β = 0.95; T = 16°C (61°F)

$$SAOR = 39.5\ (kgO_2/d) \times \frac{9.02\ (mg/L) \times 1.024^{(20-16)}}{0.90 \times [0.95 \times 9.01\ (mg/L) - 2.0\ (mg/L)]}$$

$$= 66.4\ kg\ O_2/d\ (146\ lb\ O_2/d)$$

$$Aerator\ Power\ Requirements\ (kW) = \frac{SAOR\ (kgO_2/d)}{O_2\ Transfer\ Rate\ (kgO_2/kW-d)}$$

$$Aerator\ Power\ Requirements = \frac{66.4\ (kgO_2/d)}{37\ (kgO_2/kW-d)} = 1.80\ kW\ (2.4\ hp)$$

Since blowers typically have an efficiency of 50% or less, select two aerators with 3.73 kW (5 hp) motors or two 2-speed aerators with 7.5/5 hp motors per basin normally operated on low speed.

c. Clarifier Diameter

Design flow = 387 500 L/d

Overflow rate = 16.3 $m^3/m^2/d$

$$Clarifier\ Area\ (m^2) = \frac{Design\ Flow\ (m^3/d)}{Overflow\ Rate\ (m^3/m^2/d)}$$

$$Clarifier\ Area = \frac{378.5\ (m^3/d)}{16.3\ (m^3/m^2/d)} = 23.2\ m^2\ (250\ ft^2)$$

$$Clarifier\ Surface\ Area = \frac{\pi D^2}{4} = 23.2\ m^2$$

Solve for diameter (D):

$$D = \sqrt{23.2/0.785} \cong 6.0\ m\ (20\ ft)$$

A new clarifier surface area is then recalculated:

$$New\ Clarifier\ Area\ (m^2) = \frac{\pi \times D^2}{4}$$

$$New\ Clarifier\ Area = \frac{\pi \times [6\ (m)]^2}{4} = 28.3\ m^2\ (305\ ft^2)$$

Detention time (hr) = Volume (m^3) × 24 (hr/d)/Flow (m^3/d)

$$Volume = Area\ (m^2) \times Depth\ (m) = 28.3\ (m^2) \times 3.04\ (m) = 86\ m^3\ (3\ 038\ ft^3)$$

(Sheet 2 of 3)

Table E-4 (Concluded)

$$Detention\ time = \frac{86\ (m^3) \times 24\ (hr/day)}{378.5\ (m^3/d)} = 5.5\ hr$$

Total clarifier diameter = water diameter + 2 × wall thickness

Total clarifier diameter = 6 m + 2 × 0.2 m = 6.4 m (21 ft)

For a 3.04 m side water depth (SWD) for the channels

$$Volume = 378.5\ m^3\ of\ wraparound\ channels$$

$$A = \frac{\pi D_1^2}{4} - \frac{\pi D_2^2}{4} = 378.5\ m^3/SWD$$

$$0.785\ D_1^2 - 0.785\ D_2^2 = 378.5/3.04$$

$$0.785\ D_1^2 - 0.785 \times 6.4^2 = 124.5$$

D_1 = 14 m (46 ft)

(Sheet 3 of 3)

E-3. Stabilization Pond

Design a facultative stabilization pond with primary treatment (clarifier and anaerobic digester of Imhoff tank design) to be followed by secondary clarification to treat a domestic wastewater flow of 378 500 L/d (100,000 gal/d). Influent BOD_5 will be 250 mg/L. Assume a primary clarifier removes 33 percent of the influent BOD_5 (BOD_5 = 0.68 BOD_u), and influent wastewater [SO_4^{2-}] is ≤500 mg/L. Four rectangular ponds in parallel are to be constructed. The controlling winter temperature of each pond will be 4.5 °C (40 °F). Length to width ratio of each pond will be 3:1, as is typical for such facilities.

Find, as shown in Table E-5:

Total area of ponds.

Applied BOD_5 loading.

Dimensions of the ponds.

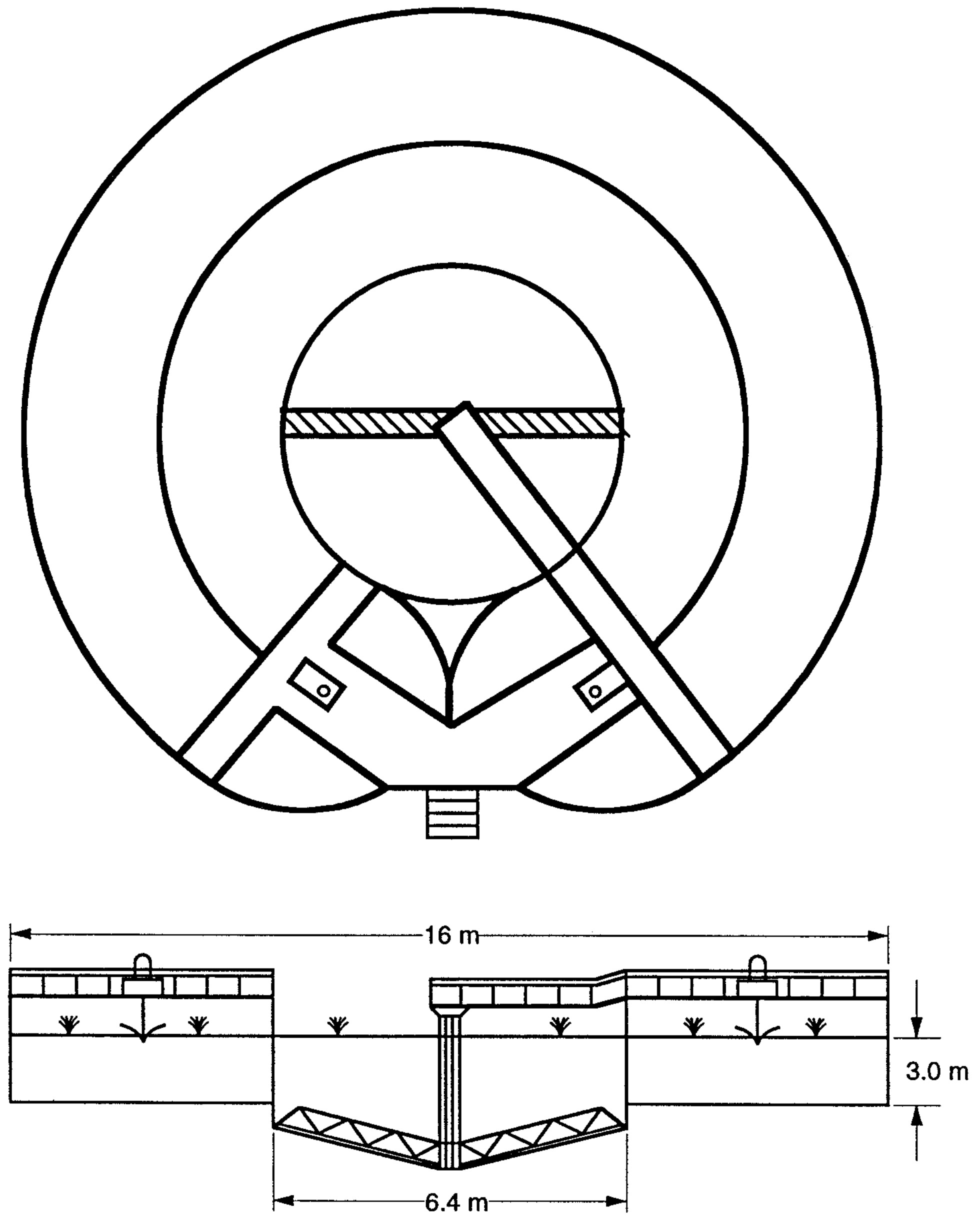

Figure E-2. Oxidation ditch carrousel wraparound (closed-loop reactor)

Table E-5
Stabilization Pond Design Calculations

BOD_5 (primary clarifier effluent) = 250 (mg/L) × (1-0.33) = 167.5 mg/L

$S_i = BOD_u$ = 167.5/0.68 = 246.3 mg/L

$$t = \frac{V}{Q} = 0.035 \times 246.3 \times [1.085^{(35-4.5)}] \times 1 \times 1 = 104\ days$$

$$V\ (4\ basins) = Q\ (m^3/d) \times t\ (d) = 378.5\ (m^3/d) \times 104\ (d) = 39\ 364\ m^3\ (31.9\ acre-ft)$$

$$A\ (4\ basins) = \frac{V}{d}$$

where:

A = total ponds area (hectares)
d = facultative pond effective depth = 1.8 m (6 ft); to include 0.3 m (1 ft) for sludge storage

$$A\ (4\ basins) = \frac{39\ 364\ (m^3)}{1.8\ (m)} \times \frac{1\ ha}{10\ 000\ (m^2)} = 2.19\ ha\ (5.41\ ac)$$

$$BOD_5\ Load = 387.5 \times 10^3\ (L/d) \times 167.5\ (mg/L) \times 10^{-6}\ (kg/mg) = 64.9\ kg/d\ (142.9\ lb/d)$$

$$\text{Applied Load} = 64.9\ (kg/d) \times \frac{1}{2.19\ ha}$$

Applied Load = 29.6 kg BOD_5/ha-d (26.5 lb BOD_5/acre-d)

$$Each\ Pond\ Area = \frac{Total\ Pond\ Area\ (ha) \times 10\ 000\ (m^2/ha)}{4} = 5\ 475\ m^2\ (17\ 958\ ft^2)$$

Pond Area = Length ×Width

Length = 3 × width (W)

Pond Area = (3W) × W = 5 475 m^2

Width (W) = 42.7 m (140 ft)

Length = 128 m (420 ft)

Use the Gloyna equation ("Facultative Waste Stabilization Pond Design in Ponds as a Wastewater Treatment Alternative," by E. F. Gloyna, J. F. Malina, Jr. and E. M. Davis):

$$t = \frac{V}{Q} = CS_i\ [\theta^{(35-T)}]ff'$$

where

V = pond volume, m^3.

C = conversion coefficient = 0.035 (a constant metric conversion).

Q = flow (378.5 m^3/d).

S_i = ultimate influent BOD_5 mg/L.

f' = sulfide or immediate chemical oxygen demand = 1 (for[SO_4^{2-}] concentrations $\leq$500 mg/L).

f = algae toxicity factor = 1.

θ = temperature coefficient (the value of θ ranges from 1.036 to 1.085, and 1.085 is recommended as it is conservative).

T = average water temperature for the pond during winter months, $^\circ$C.

t = hydraulic detention time (days).

E-4. Zero Discharge or Water Recycle/Reuse For Toilet Flush Water in Rest Areas (Closed-Loop Reuse)

a. *Background.*

(1) A combination of an extended aeration-activated sludge wastewater treatment system followed by mixed-media pressure filtration has been successful in treating liquid waste from a comfort facility with eight water closets and two urinals, plus lavatories. The design wastewater flow is 37 800 L/d (10,000 gal/d).

(2) The closed-loop reuse principle is generally applicable where liquid discharges from a recreational area are not permitted or desired. After the system is initially filled and operational, a fraction of the treated wastewater (about 6 percent) is fed to the terminal holding pond or lagoon to evaporate to account for the makeup water used for lavoratories and drinking fountains. The makeup water is estimated to represent about 6 percent of total water use. The sludge from the waste solids holding basin is periodically removed by tank truck. The design parameters for the original extended aeration treatment system are presented in Table E-6. Figure E-3 presents a schematic flow diagram of the wastewater recycle-reuse system.

(3) It is to be expected that 90 to 95 percent of water used in a comfort station facility is for the water closets or toilet flushing functions. Generally, 10 to 20 cycles are required for the system to reach equilibrium with an input of 5 to 10 percent of potable water for the lavatories or drinking fountains. The wastewater from lavatories and drinking fountains is considered “new” water and is a factor in the control of the amount of wastewater that must be fed to the final holding pond to evaporate.

(4) Operating records reveal no objectionable odors from the water closets or lavatories, no objectionable colors from blue (or other food dyes) introduced to give a sanitized look to the flushing waters, no foaming in the sanitary facilities, and no building of total suspended solids. The 90 to 95 percent of reused water in the water closets and urinals has an acceptable quality following the extended aeration process and multimedia filtration.

(5) Use surveys to indicate that toilet flush water use is about 12.7 L (3.4 gal) per flush and 15.0 L (4.0 gal) per toilet user. Potable water use (lavatories and drinking fountains) is approximately 0.8 L (0.2 gal) per toilet user. Average resident time in the toilet facility is expected to be 3 min.

b. *Recycled wastewater.*

(1) The desired treatment characteristics of the recycled wastewater are shown in Table E-7.

Table E-6
Original Design Parameters of the Existing Extended-Aeration System

Parameter	Value
Design flow:	37 800 L/d (10,000 gal/d)
Aeration Tank:	
Detention time	24 hr
Volume	38 m^3 (1342 ft^3)
Oxygen transfer rate	756 g/hr (1.7 lb/hr)
Max. Return Solids flow:	1.83 L/s (0.48 gal/s)
Waste solids holding basin:	15.1 m^3 (533 ft^3)
Comminutor:	8.8 L/s (2.3 gal/s)
Settling basin:	
Detention time	4 hr
Volume	6.3 m^3 (222 ft^3)
Holding pond:	
Volume	567 m^3 (20,000 ft^3)
Surface area	497 m^2 (5350 ft^2)

The comminutor shreds to 6 mm (0.2 in). An overflow bypass around the comminutor has a manually cleaned medium bar screen. All pumps are pneumatic.

Table E-7
Desired Characteristics of Recycled Wastewater

Parameter	Concentration Range
MLSS	3000 to 5000 mg/L
Settleability	200 to 850 mL
(as determined by the MLSS volume in 1 L graduated cylinder after 1 hr)	
Alkalinity	
pH	
TSS	50 to 500 mg/L
	5.5 to 8.3
	< 15 mg/L

(2) To achieve the best operation, the recycled wastewater must be chemically stable and the total suspended solids and total volatile solids must remain relatively constant. The most desirable range for MLSS would probably be 3500 to 4000 mg/L with an accompanying settleability of 400 to 600 mL.

c. Unit processes for closed-loop reuse.

(1) The unit processes shown in Table E-8 have been added for the closed-loop reuse to meet the desired characteristics identified in Table E-7.

(2) The multimedia rapid filtration pressurized vessel has a design filtration rate of 80 to 160 L/min/m^2 and a backwash design flow rate of 285 to 610 L/min/m^2. The filter appears to operate best at a filtration rate of 94 L/min/m^2 (2.3 gal/min/ft^2) and at a backwash cleaning rate of 345 L/min/m^2 (8.5 gal/min/ft^2). Total suspended solids in the recycled wastewater must be less than 15 mg/L for reuse in the toilet facility.

E-5. Sequencing Batch Reactor (SBR)

a. General.

(1) The design of a sequencing batch reactor (SBR) involves the same factors commonly used for the flow-through activated sludge system. The aspects of a municipally treated waste which require

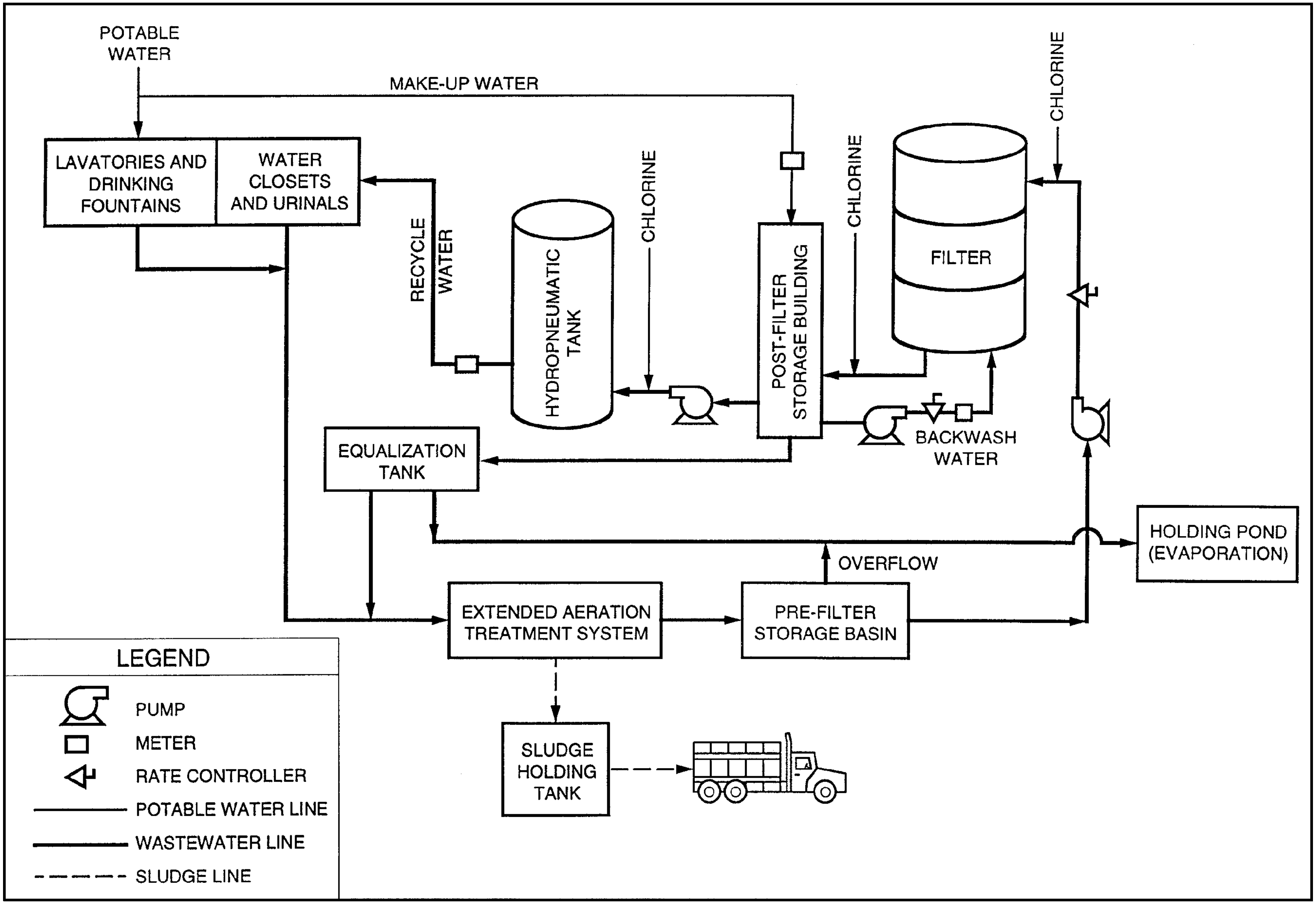

Figure E-3. Flow diagram for wastewater recycle-reuse system

Table E-8
Unit Processes for the Closed Loop System

Unit Process	Design Parameter
Pressure Filter	
Diameter	1.8 m (6 ft)
Media	Granular nonhydrous aluminum silicate
	(Effective size = 0.57,
	Uniform coefficient = 1.66)
	4.1 L/s (65 gal/min)
Max. pump rate	1.6 L/s/m^2 (0.04 gal/s/ft^2)
Filtration rate	2.65 L/s/m^2 (0.06 gal/s/ft^2)
Surface wash rate	5.77 L/s/m^2 (0.13 gal/s/ft^2)
Backwash rate	
Pre-Filter Storage Tank	75.6 m^3 (2670 ft^3)
Post-Filter Storage Tank	75.6 m^3 (2670 ft^3)
Equalization Tank	18.9 m^3 (668 ft^3)
Hydropneumatic Tank	
Total volume	18.9 m^3 (668 ft^3)
Operating volume	5.3 m^3 (187 ft^3)

dentrification as well as nitrification plus biological phosphorous removal need additional design considerations. Pretreatment of the wastewater before influent in the SBR reactor system is also required.

(2) The following example should be considered an outline to identify reactor volume elements, a diffused aeration system, the basis for signing effluent decanter units, and waste sludge systems for a system receiving 378 500 L/d (100,000 gal/d) of wastewater.

(3) Food-to-mass (F/M) ratio typically ranges from 0.05 to 0.30 with domestic waste F/M ratios typically ranging from 0.10 to 0.15. At the end of the decant phase, the MLSS concentration may vary between 2000 and 5000 mg/L. A typical value for a municipal waste would be 3500 mg/L. The MLSS concentration changes continuously throughout an SBR operating cycle from a maximum at the beginning of a fill phase to a minimum at the end of the react phase.

b. *Reactor volume.* Calculate the reactor volume based on the desired BOD_5 removal, the F/M ratio, and the MLSS. The F/M ratio and the MLSS at the low water level determine the reactor volume at the low water level, as shown in Table E-10.

c. *Decant volume.* Calculate the decant volume as the difference between the reactor volume and the low water volume, as shown in Table E-10. Each operating cycle is normally composed of *mixed fill*, *react fill*, *settle*, *decant*, *sludge waste*, and *idle*. The number of cycles dictates the number of decants per day or the volume of liquid to be decanted for each cycle. The volume per decant per cycle must be selected based on the maximum sustained daily flow.

d. *Detention time.* Calculate the maximum detention time based on the reactor volume. Calculate the minimum detention time based on the decant volume, as shown in Table E-10.

e. *SBR dimensions.* Estimate the required unit process dimensions, as shown in Table E-10. The basin length can be estimated based on a recommended minimum depth. The minimum depth after decant is determined as the depth of a clarifier in a flow-through system, i.e., quiescent settling and a large settling area. A minimum depth of 2.75 m (9 ft) is typically recommended by designers.

Table E-9
Design Assumptions

Influent/Effluent Composition

Parameter	Influent	Effluent
BOD5	250 mg/L	25 mg/L
TSS	250 mg/L	30 mg/L
NH3-N	25 mg/L	1 mg/L
Total Phosphorous	10 mg/L	2 mg/L
TKN	40 mg/L	5 mg/L

Assumptions

F/M Ratio	
($kgBOD_5$applied/kgMLSS-d):	0.13
MLSS:	3500 mg/L
Minimum clarifier depth:	2.75 m (9 ft)
Net sludge yield (kg MLSS/kg BOD_5):	
Wastewater temperature:	0.76
Minimum solids retention time:	$20^oC(67^oF)$
Reactor volume decanted each day:	8 d
Net elevation above sea level:	60%
DO mixed liquor concentration (C_O):	304 m (1000 ft)
Oxygen coefficients:	2 mg/L
kg O_2/kg BOD_5	
kg O_2/kg NH_3-N	1.28
Transfer factors:	4.60
α (typical for coarse bubble diffuser)	
β (tyical for domestic wastewater)	0.85
Typical O2 transfer rate for coarse bubble diffusers:	
	0.95
Number of cycles per day:	
Include two square basins for operational flexibility	1.25 kgO_2/kW-hr
	(2 lbO_2/hp-hr)
	4

f. *Aeration power.* Calculate the blower capacity based on the sludge production rate and the total oxygen demand, as shown in Table E-10. In sizing the aeration equipment, it must be noted that the equipment operates only a portion of the SBR operating cycle (part of the fill plus react phases). Therefore, the total daily oxygen requirements must be met in a shorter time period than in a conventional activated sludge flow-through system. The total daily oxygen requirements are estimated by adding the carbonaceous oxygen demand (oxygen required for BOD_5 oxidation) to the nitrogeneous oxygen demand (oxygen required for TKN oxidation).

g. *Sludge and decanter flows.* Calculate the sludge and decanter flow rates at design conditions, as shown in Table E-10.

h. *Equipment specifications*. Figure E-4 presents a plan view of the SBR system with the following specifications:

(1) At least two basins are provided in an SBR design to provide operational flexibility and improved effluent quality. SBR unit dimensions:

maximum volume = 470 m^3 (16,598 ft^3).
high water level = 5.80 m (19 ft).
maximum decant height = 3.05 m (10 ft).
low water level = 2.75 m (9 ft).
hydraulic detention time at low water level = 17.9 hrs.

Table E-10
SBR Design Calculations

a. Reactor Volume

$$BOD_5\ Removed\ (kg/d) = [(BOD_{influent} - BOD_{effluent})\ (mg/L)] \times Flow\ (L/d) \times 10^{-6}\ (kg/mg)$$

$$BOD_5\ Removed = (250 - 25) \times 378.5 \times 10^{-3} = 85.2\ kg/d\ (187\ lb/d)$$

$$Required\ Aerobic\ Mass\ (kg) = \frac{BOD_5\ Removed\ (kg/d)}{F/M\ Ratio\ (kgBOD_5/kbMLSS{-}d)}$$

$$Required\ Aerobic\ Mass = \frac{85.2\ (kgBOD_5/d)}{0.13\ (kgBOD_5/kgMLSS{-}d)} = 656\ kg\ MLSS$$

$$Reactor\ Volume_{(low\ water\ level)}\ (m^3) = \frac{MLSS\ Mass\ (kg)}{MLSS\ Concentration\ (mg/L)} \times \frac{10^6\ (mg/kg)}{10^3\ (L/m^3)}$$

$$Reactor\ Volume_{(low\ water\ level)} = \frac{656\ (kg)}{3\ 500\ (mg/L)} \times 10^3\ (mg{-}m^3/kg{-}L) = 188\ m^3\ (6\ 634\ ft^3)$$

Since the decant volume represents 60% of the total volume,

Total Reactor Volume = 188/(1-0.6) = 470 m^3 *(16 598* ft^3*)*

b. Decant Volume

Total Decant Volume = Total Reactor Volume (m^3) *- Reactor* $Volume_{(low\ water\ level)}$ (m^3)

Total Decant Volume = 470 (m^3) *- 188* (m^3) *= 282* m^3 *(9 959* ft^3*)*

c. Detention Time

$$Max.\ Detention\ Time\ (hr) = \frac{Total\ Reactor\ Volume\ (m^3)}{Flow\ (L/d) \times 10^{-3}\ (m^3/L)} \times 24\ (hr/d)$$

$$Max.\ Detention\ Time = \frac{470\ (m^3) \times 24\ (hr/d)}{378.5 \times 10^3\ (L/d) \times 10^{-3}\ (m^3/L)} = 30\ hr$$

$$Min.\ Detention\ Time\ (hr) = \frac{Decant\ Volume\ (m^3)}{Flow\ (L/d) \times 10^{-3}\ (m^3/L)} \times 24\ (hr/d)$$

$$Min.\ Detention\ Time = \frac{282\ (m^3) \times 24\ (hr/d)}{378.5 \times 10^3\ (L/d) \times 10^{-3}\ (m^3/L)} = 17.9\ hr$$

d. SBR Dimensions

$$Basin\ Area\ (m^2) = \frac{Basin\ Volume_{low\ water\ level}\ (m^3)}{Minimum\ Depth\ (m)} = \frac{188\ (m^3)}{2.75\ (m)} = 68.4\ m^2\ (736\ ft^2)$$

$$Basin\ Length = \sqrt{68.4\ (m^2)} \cong 9\ m\ (30\ ft)$$

(Sheet 1 of 3)

Table E-10. (Continued)

$$Basin\ Depth\ (m) = \frac{Total\ Reactor\ Volume\ (m^3)}{Basin\ Area\ (m^2)} = \frac{470\ (m^3)}{[9\ (m)]^2} = 5.8\ m\ (19\ ft)$$

e. Aeration Power

$$Nitrogeneous\ O_2\ Demand\ (kg\ O_2/d) = NH_3\text{-}N_{oxidized}\ (kg/d) \times kg\ O_2/kg\ BOD_5$$

$$NH_3\text{-}N_{oxidized}\ (kg/d) = TKN\ Removed\ (kg/d) - Synthesis\ N\ (kg/d)$$

$$TKN\ Removed = (40 - 5) \times 378.5 \times 10^{-3} = 13.25\ kg/d\ (29\ lb/d)$$

Synthethis N = 5% waste activated sludge of total daily sludge production

$$Sludge\ Production\ (kg/d) = Net\ Sludge\ Yield\ (kgMLSS/kgBOD_5) \times BOD_5\ Removed\ (kg/d)$$

$$Sludge\ Production = 0.76\ (kgMLSS/kgBOD_5) \times 85.2\ (kg/d) = 64.8\ kg/d\ (143\ lb/d)$$

$$Synthethis\ N = 0.05 \times 64.8\ (kg/d) = 3.24\ kg/d\ (7\ lb/d)$$

$$NH_3\text{-}N_{oxidized} = 13.25\ (kg/d) - 3.24\ (kg/d) = 10\ kg/d\ (22\ lb/d)$$

$$Nitrogeneous\ O_2\ Demand = 10\ (kgNH_3\text{-}N_{oxidized}/d) \times 4.6\ (kgO_2/kgNH_3\text{-}N_{oxidized}) = 46\ kgO_2/d$$

$$Carbonaceous\ O_2\ Demand\ (kg\ O_2/d) = BOD_5\ Mass\ (kg/d) \times kg\ O_2/kg\ BOD_5$$

$$Carbonaceous\ O_2\ Demand = 3.24\ (kgBOD_5/d) \times 1.28\ (kgO_2/kgBOD_5) = 4.15\ kgO_2/d$$

AOR (kg/d) = Carbonaceous O2 Demand (kg/d) + Nitrogeneous O2 Demand (kg/d)

AOR = 4.15 kgO_2/d + 46 kgO_2/d = 50.15 kgO_2/d

where:

AOR = Actual Oxygen Requirements (kg O_2/d)

$$SAOR\ (kg\ O_2/hr) = \frac{AOR \times C_S \times \Theta^{(T - 20)}}{\alpha \times (\beta \times C_{SW} - C_O) \times Blower\ Usage\ (hr/d)}$$

where:

SAOR = Standard Actual Oxygen Requirements (kg O_2/d)
Θ (temperature correction factor) = 1.024
C_S (O_2 saturation concentration at standard temperature and pressure) = 9.02 mg/L
C_{SW} = concentration correction for elevation (i.e., 1 000 ft) = 9.02 - 0.0003 × elevation
C_{SW} = 9.02 - 0.0003 × 1 000 = 8.72 mg/L (NOTE: 0.0003 may be used as a rule-of-thumb describing a 0.0003 mg/L rise/drop in DO saturation concentration per every foot of elevation increase/decrease.)
C_O = 2 mg/L
α = 0.85; β = 0.95; T = 20°C (67°F)
Blower Usage = 14 hr/d (based on 4 cycles per day (6 hr/cycle), 1.0 hr fill time, 3.5 hr react time, 0.75 hr settle time, 0.5 hr decant time, and 0.25 hr idle time)

(Sheet 2 of 3)

Table E-10. (Continued)

$$SAOR = \frac{50.15\ (kgO_2/d) \times 9.02\ (mg/L) \times 1.024^{(20-20)}}{0.85 \times (0.95 \times 8.72 - 2)\ (mg/L) \times 14\ (hr/d)} = 6.1\ kgO_2/hr\ (13.4\ lbO_2/hr)$$

$$Motor\ Requirements\ (kW) = \frac{SAOR\ (kgO_2/d)}{O_2\ Transfer\ Rate\ (kg/kW{-}d)}$$

$$Motor\ Requirements = \frac{6.1\ (kgO_2/hr)}{1.25\ (kgO_2/kW{-}hr)} = 4.9\ kW\ (6.5\ hp)$$

Since blowers typically have an efficiency of 50% or less, select two aerators with 11.2 kW (15 hp) motors. Blower size depends on the standard air flow rate. The standard air flow rate in standard cubic meters per minute (SCMM) is calculated as follows:

$$SCMM = \frac{\dfrac{SAOR\ (kgO_2/d)}{Blower\ Usage\ (hr/d)} \times \dfrac{1\ hr}{60\ min}}{O_2\ Content\ (kgO_2/m^3\ air) \times Absorption\ Efficiency}$$

where:
Air O_2 Content (at standard conditions) = 0.2793 kg O_2/m^3 of air
Obtain blower absorption efficiency from manufacturers

f. Sludge and Decant Flows

$$Sludge\ flow\ rate\ (L/d) = \frac{Sludge\ Mass\ Flow\ (kg/d)}{Sludge\ Density\ (kg/L)}$$

Typical sludge density = 1.02 kg/L

$$Decanter\ Flow\ Rate\ \ (L/\text{min}) = \frac{MDF}{NB \times NCB \times MCT}$$

where:
MDF = maximum daily flow for decant (or sludge waste)
NB = number of basins
NCB = number of cycles per basin
MCT = maximum cycle time for de cant or sludge waste (min)

(Sheet 3 of 3)

(2) Blowers: rotary positive displacement.

(3) Diffusers: 4-10 tube coarse bubble retriever diffuser assembly (2 per basin).

(4) Mixers: 2 at 3.73 kW (5 hp).

(5) Sludge pumps: 2 at 1.49 kW (2 hp).

(6) Decanter sizing: cycles per day = 4.
volume per decant = 70.5 m^3 (2490 ft^3).
decant time = 30 min.
decant flow rate = 2.35 m^3/min (621 gal/min).

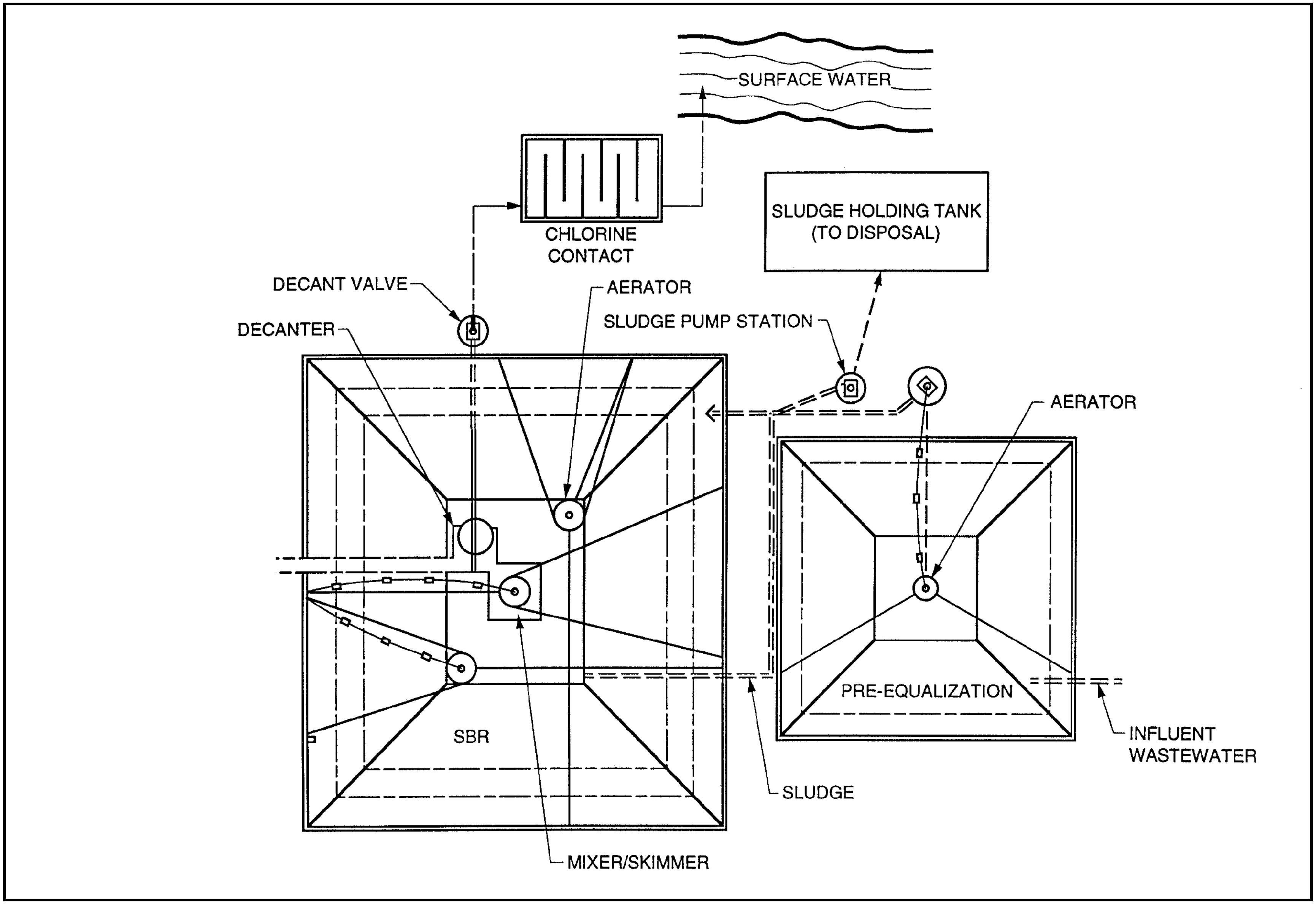

Figure E-4. Sequencing batch reactor

Table E-11
Design Assumptions

Influent/Effluent Composition		
Parameter	**Influent**	**Effluent**
BOD_5	250 mg/L	< 30 mg/L
TSS	250 mg/L	< 30 mg/L
TKN	20 mg/L	< 5 mg/L
Total Phosphorous	10 mg/L	
Winter Temperature	20^oC (67^oF)	
Assumptions		
L:W Ratio (for all basins):		
Hydraulic Loading:	3:1	
	200-600 m^3/ha-d	
BOD_5 Loading:	(20,000-65,000 gal/acre-d)	
	40-80 kg/ha-d	
TKN Reduction:	(36-71 lb/acre-d)	
	70-90%	

(7) Influent valves: 2, each 150 mm (6 in.) diameter.

(8) Air blower values: 2, each 150 mm (6 in.) diameter.

E-6. Constructed Wetlands Aerobic Non-Aerated Hyacinth System

Design an aerobic non-aerated hyacinth constructed wetlands secondary treatment system for a municipal wastewater flow of 284 000 L/d (75,000 gal/d). The system will require preliminary or pretreatment (Imhoff tank). Disinfection of effluent may be required depending upon regulatory restrictions.

a. BOD_5 loading. Calculate the influent BOD_5 loading using the influent BOD_5 concentration and the design flow, as shown in Table E-12.

b. Basin surface area. Calculate the required basin surface area at moderate BOD_5 loading rate of 50 kg/ha-d and the required area for the primary or first cells at a BOD5 loading rate of 100 kg/ha-d, as shown in Table E-12.

c. Primary cell dimensions. Use two primary cells. Calculate the dimemsions of the two primary cells, as shown in Table E-12.

d. Final cell dimensions. Use four final cells. Calculate the dimensions of the four final cells, as shown in Table E-12.

e. Cell volume. Calculate the primary and final cells volume, as shown in Table E-12. Allow 0.5 m (1.6 ft) for sludge storage and assume 1.2 m (4 ft) effective water depth for a total pond depth of 1.7 m. Use a 3:1 sideslopes ratio to determine the treatment volume (approximation of a frustrum).

f. Hydraulic detention time. Estimate the hydraulic detention time in the effective, or above-sludge, level (zone) in the primary and final cells, as shown in Table E-12.

g. Hydraulic loading. Check the hydraulic loading to ensure that a minimum of 75 percent total nitrogen reduction is achieved to comply with the effluent quality requirements, as shown in Table E-12.

Table E-12
Constructed Wetlands Aerobic Non-Aerated Hyacinth System

a. BOD_5 Loading

$$BOD_5\ Loading\ (kg/d) = Influent\ BOD_5\ (mg/L) \times Flow\ (L/d) \times 10^{-6}\ (kg/mg)$$

$$BOD_5\ Loading\ (kg/d) = 250\ (mg/L) \times 284 \times 10^3\ (L/d) \times 10^{-6}\ (kg/mg) = 71\ kg/d\ (157\ lb/d)$$

b. Basin Surface Area

$$Total\ Area\ Required\ (ha) = \frac{BOD_5\ Loading\ (kg/d)}{Moderate\ Loading\ Rate\ (kg/ha-d)}$$

$$Total\ Area\ Required = \frac{71\ (kg/d)}{50\ (kg/ha-d)} = 1.42\ ha\ (3.5\ acre)$$

$$Area\ Primary\ Cells\ (ha) = \frac{BOD_5\ Loading\ (kg/d)}{Primary\ Loading\ Rate\ (kg/ha-d)}$$

$$Area\ Primary\ Cells = \frac{71\ (kg/d)}{100\ (kg/ha-d)} = 0.71\ ha\ (1.75\ acre)$$

c. Primary Cells Dimensions

$$Cell\ Area\ (ha) = \frac{Area\ Primary\ Cells\ (ha)}{2} = L \times W = L \times \frac{L}{3}$$

$$Cell\ Area = \frac{0.71\ (ha)}{2} \times 10\ 000\ (m^2/ha) = \frac{L^2}{3}$$

L = 103 m (338 ft)
W = 34 m (112 ft)

d. Final Cells Dimensions

Divide the remaining required area (1.42 ha - 0.71 ha = 0.71 ha) into two sets of two basins each (four cells of 0.18 ha each) to produce a total system with two parallel sets with three basins each.

$$Area\ Final\ Cells = 0.18\ (ha) \times 10\ 000\ (m^2/ha) = \frac{L^2}{3}$$

L = 73.5 m (241 ft)
W = 24.5 m (80 ft)

e. Cells Volume

$$V = [(L)(W) + (L - 2sd)(W - 2sd) + 4(L - sd)(W - sd)] \times \frac{d}{6}$$

(Continued)

Table E-12 (Concluded)

where:
- V = basin volume (m^3)
- L = basin length at surface area (m)
- W = basin width at surface area (m)
- d = effective water depth = 1.2 m
- s = slope factor (e.g., 3:1 slope, s = 3)

V (primary cell) = 3631 m^3 (128,228 ft^3)

V (final cell) = 1758 m^3 (62,083 ft^3)

f. Hydraulic Detention Time

$$Detention\ Time\ (d) = \frac{No.\ Cells \times Unit\ Volume\ (m^3)}{Flow\ (m^3/d)}$$

$$Detention\ Time\ (Primary)\ t_p = \frac{2 \times 3\ 631\ (m^3)}{284\ (m^3/d)} = 26\ days$$

$$Detention\ Time\ (Final)\ t_p = \frac{4 \times 1\ 758\ (m^3)}{284\ (m^3/d)} = 25\ days$$

Total detention time = 26 + 25 = 51 days at 20° C (67°F)

g. Hydraulic Loading

$$Hydraulic\ Loading = \frac{284\ m^3/d}{1.42\ ha} = 200\ m^3/ha-d$$

This hydraulic loading is within the recommended range and is sufficient (i.e., ≤ 935 m^3/ha-d) to reduce the nitrogen loading by 70- 90 percent. It is reasonable to expect 5 mg/L of nitrogen or less in the effluent. Because the nitrogen will not be at optimum growth levels in this system, an annual harvest is suggested. An influent flow diffuser in each of the primary cells is recommended to properly distribute the untreated effluent.